LRFD Design Office Study

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The Proposed Load and Resistance Factor Design (LRFD) Criteria[1] were subjected to practical testing in a two-part Design Office Study. Representative steel members were designed using the AISC Specification[2] and using the LRFD Criteria for a range of building types, loads, and spans. Primary objectives of the study were to:

1. Verify acceptable LRFD results.
2. Identify needs for clarifying LRFD Criteria.
3. Evaluate routine use of LRFD in design offices.
4. Suggest possible design aids.

Engineers engaged in the Design Office Study selected actual buildings which had been designed by their firms for most of the cases studied. While making their computations, they wrote questions and comments in a log book, which later proved useful to the task committee of the sponsoring organization (American Iron and Steel Institute).

Two prominent engineering firms participated in the study: Sverdrup and Parcel Associates, for which Mr. Arthur Meenan was in charge, and LeMessurier Associates, for which the writer was in charge.

PROJECTS STUDIED

The Design Office Study included two general building types, divided into a total of five projects (see Table 1):

A. Project A was for a large, basically one-story Aircraft Repair Facility located in Texas. Comparative designs were made for a typical roof purlin, for two different welded roof trusses, and for typical interior and exterior columns, as shown in Figs. 1 and 2. The roof was up to 43 ft high above the main floor, where trusses T6 supported a mezzanine office floor. All steel was ASTM A36. Live loads were:

Roof: 20 psf
Mezzanine: 50 psf
Wind (ANSI A58.1-1972, Exp. C): 70 mph

Crane and monorail loads were included.
Both column types resisted some wind forces by strong axis bending.

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Ed. Note: This paper is based on a talk given at the 1977 AISC National Engineering Conference in Washington, D.C. It is the third of three papers on LRFD in this issue.

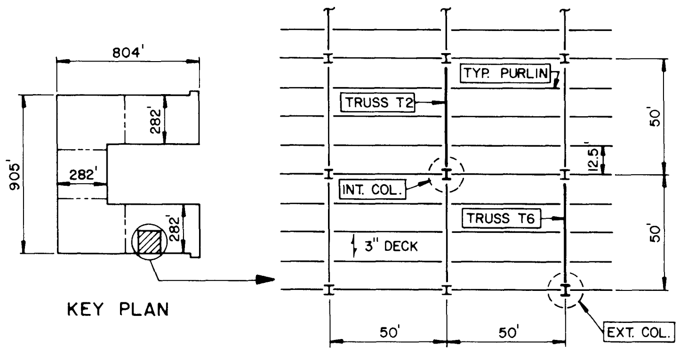


Fig. 1. Project A—Industrial building roof framing: aircraft repair facility

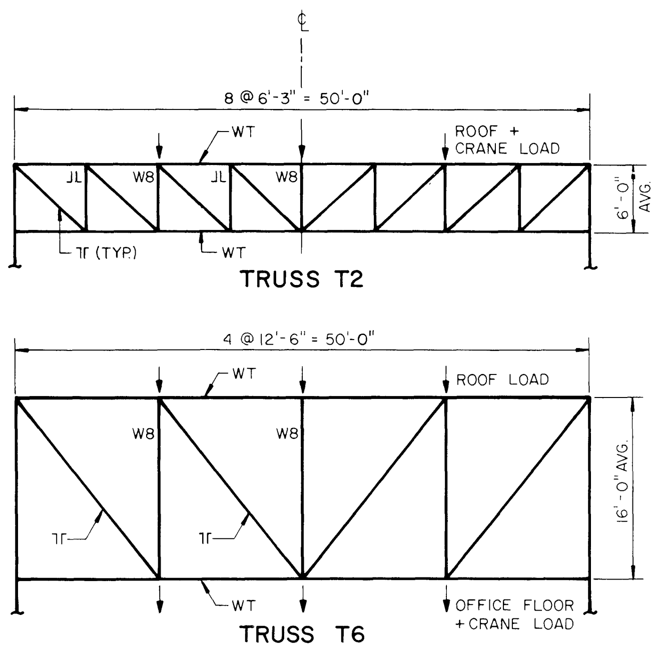


Fig. 2. Project A—Roof truss elevations

Table 1

Firm	Type		Project
Sverdrup & Parcel Associates	Industrial Structures	A	Aircraft Repair Facility—Roof Framing
		B	Brewhouse—Floor Framing
LeMessurier Associates	Commercial Structures	C	Parking Garage—Framing
		D	Mid-rise Office Building—Floor Framing
		E	Mid-rise Office Building—Wind Frames

B. Project B was to study a heavily loaded brewery brewhouse floor framed with steel beams and plate girders, as shown in Fig. 3. Floor dead load was 135 psf and live load was 300 psf. Comparative designs were made for a typical simply supported 23-ft-span beam and a 62-ft-span plate girder.

C. Project C was to study framing for a six-story passenger car parking garage, located in Boston. Floor live load was 50 psf. Story heights were 7 ft-6 in. *clear.* A simplified framing plan, Fig. 4, shows beam, girder, and column elements for which comparative designs were made. Wind resistance is provided by stair towers, not shown.

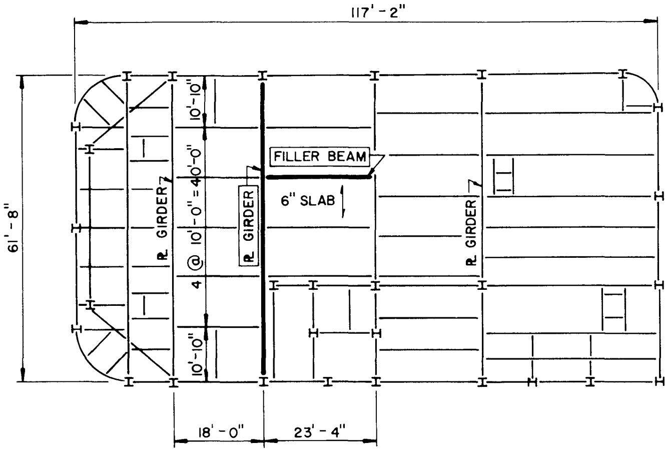


Fig. 3. Project B—Industrial building floor system: brewhouse floor framing plan

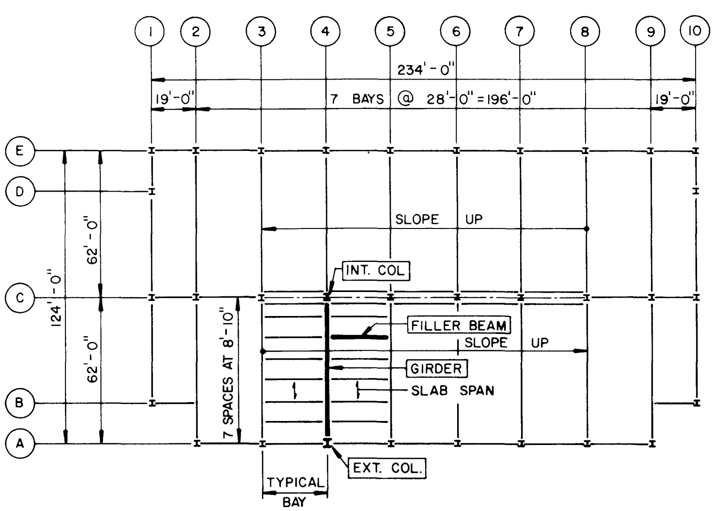


Fig. 4. Project C—Parking garage floor system: floor framing plan

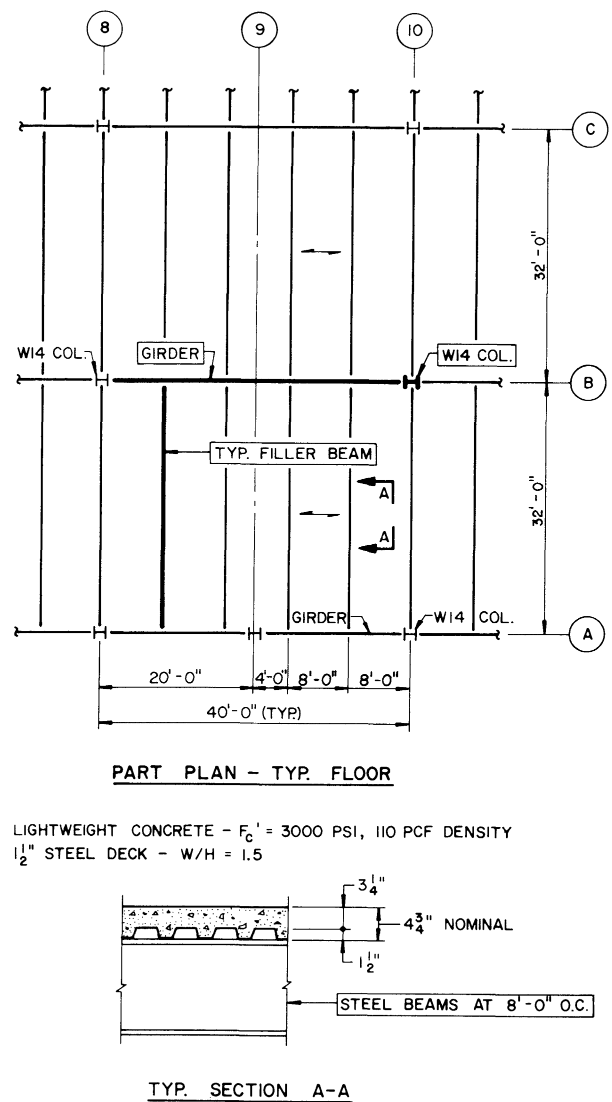


Fig. 5. Project D—Office building floor system

D. Project D was to study floor framing of a Boston 16-story office building, having $3\frac{1}{4}$-in. lightweight concrete topping over a $1\frac{1}{2}$-in. steel deck. Designs were made for a typical 32-ft-span filler beam and for a 40-ft-span girder, as shown in the part plan, Fig. 5, for three different live loads:

Case 1: 50 psf LL + 20 psf partitions
Case 2: 100 psf LL (no reduction)
Case 3: 100 psf LL (reduced)

E. Project E was to study the wind-resisting frames of the same 16-story building as in Project D. This project was divided into three "tasks," E1, E2, and E3, as schematically indicated in the floor plan of Fig. 6. Total building height was 215 ft, with typical story heights 13 ft-4 in.

Comparative designs were made for each task. The wind bent and column sizes were designed at two levels—stories 2 and 10. Design live load was 50 psf, reduced, plus 20 psf partition allowance. Wind loading used was ANSI A58.1-1972 Exposure B, 90 mph. ASTM A572 grade 50 steel was used whenever practicable.

Task E1 was to study columns and girders of an interior moment-framed wind bent in the long direction. In the as-built structure, this bent carries primarily gravity load, with most of the east-west wind taken by the two outside bents. Although less stiff than the outer bents, it resists about 13% of the total east-west wind.

Task E2 was to study web members and columns of one of three similar trussed wind bents. This bent resists 32% of the total north-south wind. Figure 7 shows in elevation the typical "W" arrangement of web members in this one-bay wide wind truss. All web members were double channels spaced $\frac{3}{4}$-in. apart.

Task E3 was to study a variation from the as-built design, such that *all* east-west wind is resisted by the two outer longitudinal wind bents, with interior girders and columns now simply connected. Note that the outer bents have columns at 20-ft spacing. Designed elements were a typical interior (moment-free) column and exterior wind bent columns and girders.

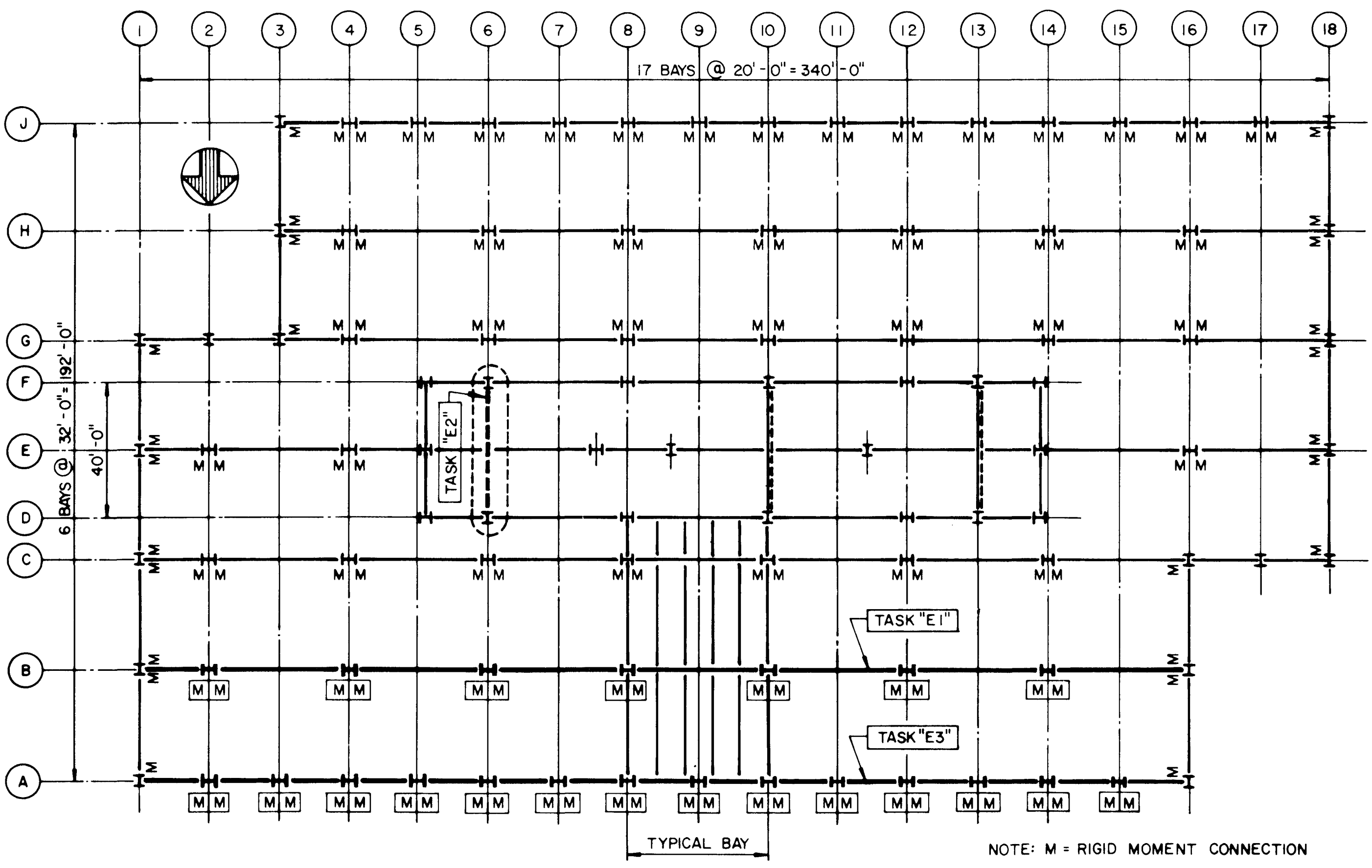


Fig. 6. Project E—Office building frames: typical framing plan

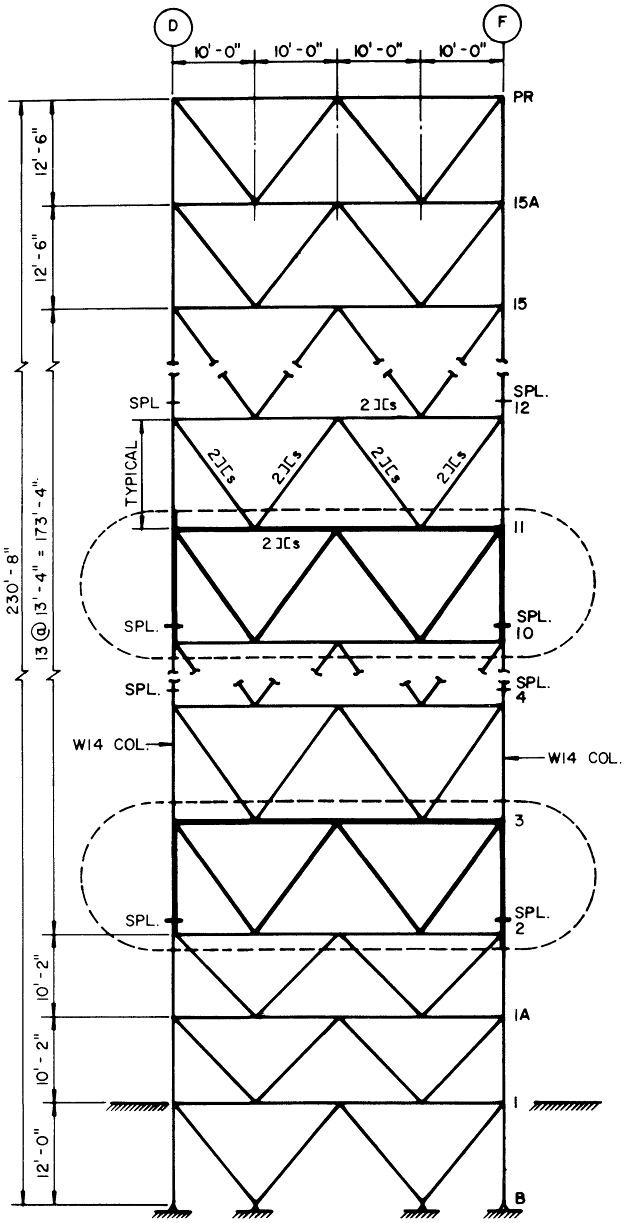


Fig. 7. Project E—Task E2, trussed wind bent

COMPARISON OF AISC AND LRFD DESIGNS

A sampling of detailed member size comparisons will be presented. These are roughly ⅔ of the Design Office Study Results, and are representative of the balance.

Project A Results—Truss T2 results are shown in Fig. 8. For this 6-ft-deep 50-ft-span truss, member size differences between AISC and LRFD designs are minor. Results for roof purlins, as well as columns, are shown in Fig. 9. The controlling case is noted, as well as the ratio of design strength to capacity. Purlin designs are the same. LRFD and AISC selected sizes for the interior and exterior col-

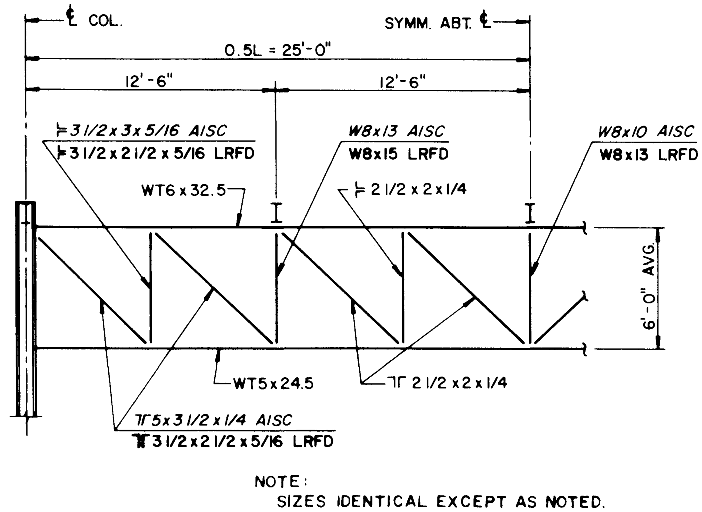
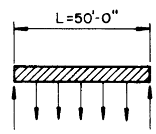


Fig. 8. Project A—Truss T2, comparative sizes



ROOF PURLINS

AISC DESIGN		LRFD DESIGN	
SIZE	*DES./CAP.*	SIZE	DES./CAP.
W21x62	*0.908*	W21x62	0.889

CONTROLLING CASE - D + L + CRANE LOADS 10 FT. O.C.

COLUMNS

COL. LOCATION	L_x	K_x	L_y	K_y	AISC DESIGN		LRFD DESIGN	
					SIZE	*DES./CAP.*	SIZE	DES./CAP
INTERIOR	26.2'	1.5	32.7'	1.0	*W14x78*	*0.973*	W14x78	1.034
EXTERIOR	27.7'	1.5	27.7'	1.0	*W12x65*	*0.850*	W12x65	0.941

CONTROLLING CASE - GRAVITY AXIAL LOAD PLUS WIND MOMENT M_x

Fig. 9. Project A—Comparative designs for roof purlins and columns

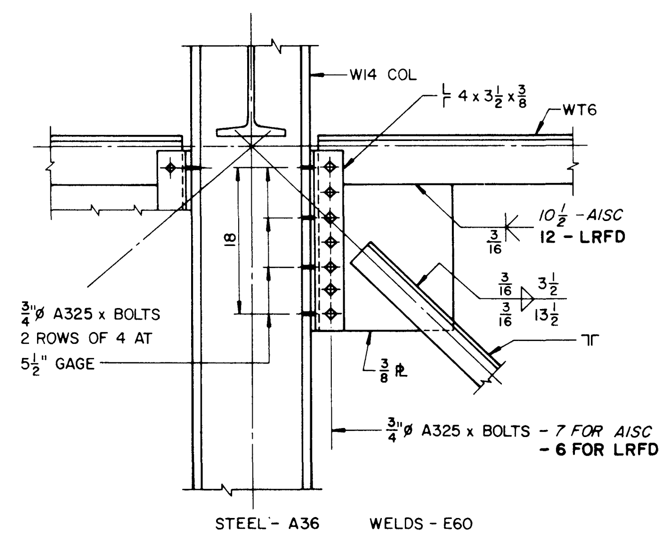
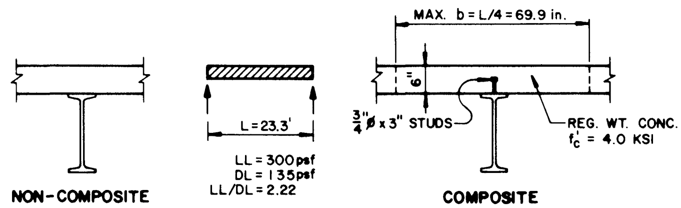


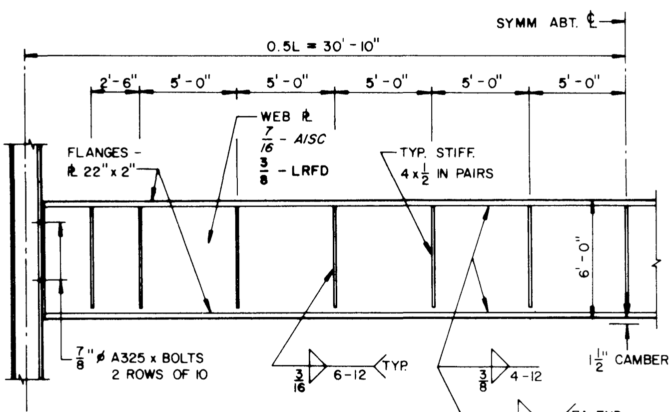
Fig. 10. Project A—Connection of truss T2 to column



AISC		LRFD	
SIZE	*DES./CAP.*	SIZE	DES./CAP.
W24x61	*1.00*	W24x61	0.98

AISC			LRFD		
SIZE	*STUDS*	*DES./CAP.*	SIZE	STUDS	DES./CAP.
W24x55	*44*	*.749*	W21x44	36	.746

Fig. 11. Project B—Filler beam comparisons



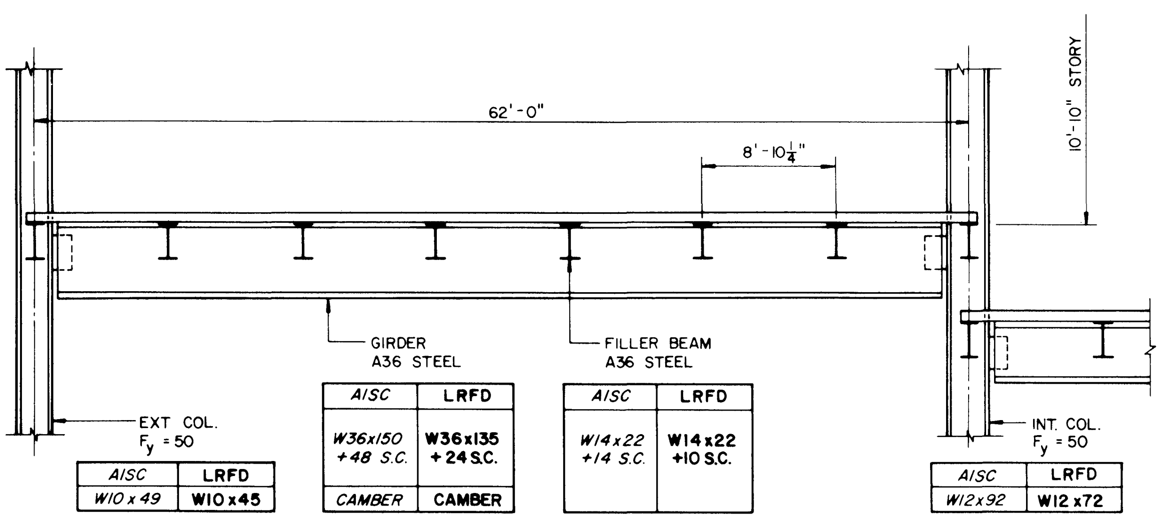
ITEM	AISC DES./CAP.	LRFD DES./CAP.
END SHEAR	*.923*	.919
MAX. MOMENT	*.977*	.973

Fig. 12. Project B—Plate girder comparisons

umns are the same, but the design/capacity ratios are lower for the AISC columns. Figure 10 shows the typical truss-column connection for truss T2. LRFD permits 6 rather than 7 bearing-type high-strength bolts.

Project B Results—Typical floor beam results are shown in Fig. 11. Alternate designs were prepared for both composite and non-composite designs. Non-composite designs are identical. Composite designs show a considerable saving for LRFD design, both in beam weight and in shear connectors. This difference could be reduced if the new 1978 AISC Specification provisions were used. Plate girder AISC and LRFD designs, shown in Fig. 12, were almost identical. For the LRFD design, identical stiffener sizes and spacings were used, but the web plate was $1/16$-in. thinner than the AISC size.

Project C Results—Figure 13 shows comparative results for the typical filler beam and the girder and column for the case of a simply connected girder. Filler beams were designed composite with the $4\frac{1}{4}$-in. concrete slab ($2\frac{1}{4}$-in. concrete on 2-in deck), and only the number of shear connectors differs. LRFD girder and column sizes are smaller than the AISC sizes, by 10% or more. Only the columns at the third story were designed.



AISC	LRFD
W10 x 49	W10 x 45

AISC	LRFD
W36x150 +48 S.C.	W36x135 +24 S.C.
CAMBER	CAMBER

AISC	LRFD
W14x22 +14 S.C.	W14x22 +10 S.C.

AISC	LRFD
W12x92	W12 x72

Fig. 13. Project C—Simple girder comparisons

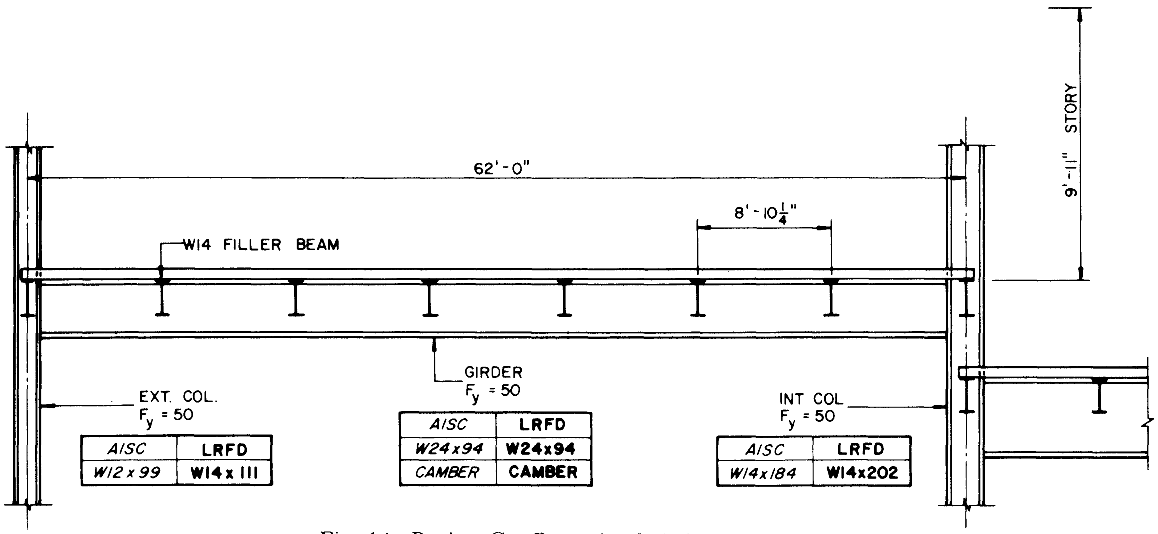


Fig. 14. Project C—Restrained girder comparisons

Figure 14 shows results for the case of a restrained, moment-connected girder. Identical girder sizes are required, but LRFD column sizes were 10 to 12% heavier. Note that the shallower restrained girder permits a story height 11 in. less than the simple girder case of Fig. 13. Simple shear connections for the filler beam and girder are shown in Fig. 15. Slotted holes in tees and A325 bolts in friction-type connections were used. The LRFD and AISC designs were almost identical.

Project D Results—Comparisons are shown in Figs. 16 and 17 for two filler beam and two simple girder cases. Loading case D2 has a higher live/dead load ratio. AISC and LRFD sizes were identical for three of the cases, with variation only in the design/capacity ratio and number of required shear studs. In girder case D3, a smaller size and fewer studs were required by LRFD, because of the lower factor on dead load and because LRFD composite design rules permit more effective use of the cross section.

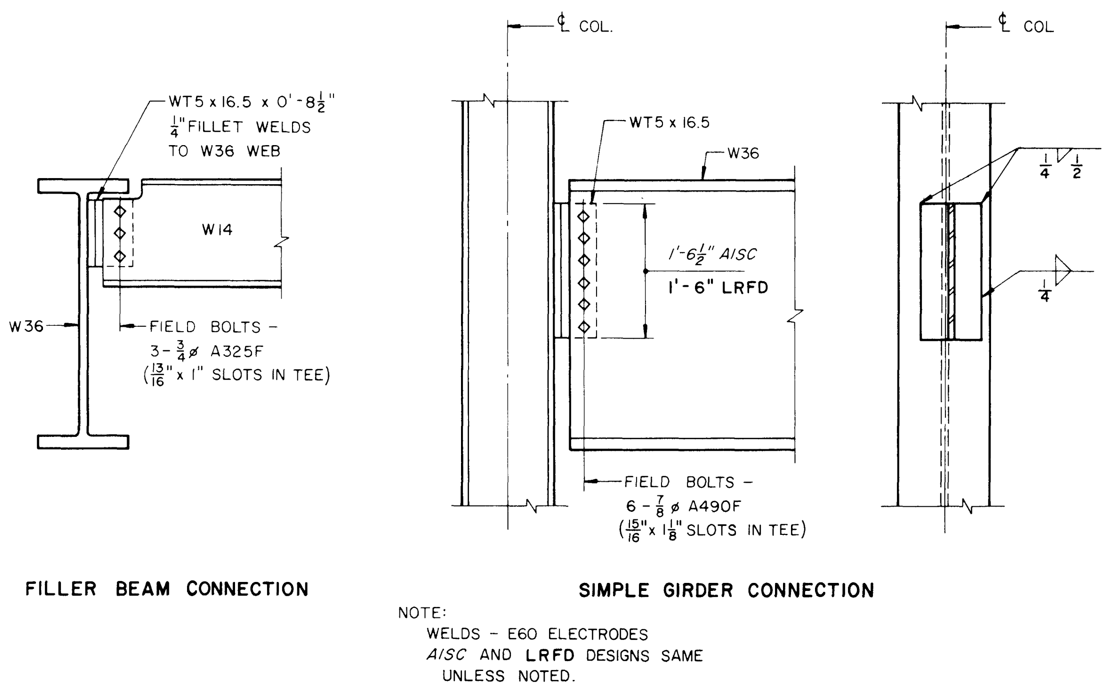
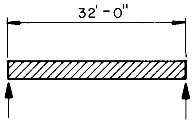
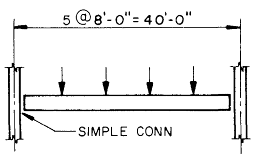


Fig. 15. Project C—Connections



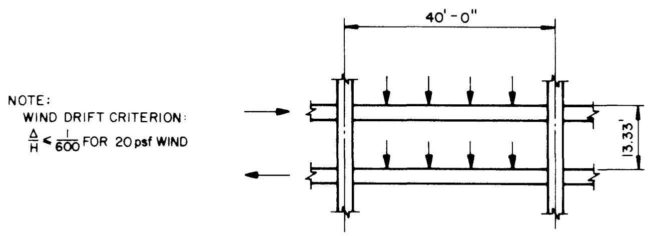
LOADING CASE	D2		D3	
DESIGN METHOD	AISC	LRFD	AISC	LRFD
BEAM DESIGN (ALL F_y = 50 ksi)	*W18x35*	W18x35	*W16x26 +14 - $\frac{3}{4}$"φ SC*	W16x26 +16 - $\frac{3}{4}$"φ S.C.
LIVE LOAD	*100 psf*	154 psf	*39.8 psf*	104.7 psf
DEAD LOAD	*52.5 psf*	63.6 psf	*74.1 psf*	65.5 psf
LL/DL RATIO	*1.90*	2.42	*0.54*	1.60
DESIGN/CAPACITY RATIO	*0.982*	0.933	*0.934*	0.752
SERVICE LOAD STRESS	*32.4 ksi*	32.4 ksi	*31.0 ksi*	30.8 ksi
SERVICE LOAD DEFL.	*.60+1.33=1.93"*	.60+1.33=1.93"	*1.07+.63=1.70"*	1.07+.63=1.70"

Fig. 16. Project D—Filler beam comparisons



LOADING CASE	D2		D3	
DESIGN METHOD	AISC	LRFD	AISC	LRFD
GIRDER DESIGN (ALL F_y = 50 ksi)	*W30x116*	W30x116	*W24x76 +42 - $\frac{3}{4}$"φ SC*	W24x68 +36 - $\frac{3}{4}$"φ S.C.
LIVE LOAD	*100 psf*	154 psf	*20.9 psf*	68.6 psf
DEAD LOAD	*56.2 psf*	68 psf	*76.5 psf*	68.1 psf
LL/DL RATIO	*1.78*	2.26	*0.27*	1.01
DESIGN/CAPACITY RATIO	*1.009*	0.961	*1.000*	0.901
SERVICE LOAD STRESS	*33.3 ksi*	33.3 ksi	*36.1 ksi*	43.3 ksi
SERVICE LOAD DEFL	*.60+1.22=1.82"*	.60+1.22=1.82"	*1.41+.70=2.11"*	1.62+.89=2.51"

Fig. 17. Project D—Simple girder comparisons



STORY	10 - 11		2 - 3	
DESIGN METHOD	AISC	LRFD	AISC	LRFD
GIRDER SIZE*	*W21x62*	W21x62	*W24x68*	W24x68
CONTROLLED BY	*GRAVITY*	GRAVITY	*WIND DRIFT*	WIND DRIFT
DESIGN/CAPACITY RATIO	*0.980*	1.010	*0.823*	0.821
MIN. SIZE FOR STRENGTH	*W21x62*	W21x62	*W21x62*	W21x62
LL/DL RATIO	*0.28*	1.01	*0.28*	1.01
COLUMN SIZE*	*W14x111*	W14x103	*W14x264*	W14x264
CONTROLLED BY	*GRAVITY*	GRAVITY	*GRAVITY*	GRAVITY
DESIGN/CAPACITY RATIO	*0.982*	0.949	*1.003*	0.927

*ALL F_y = 50 ksi

Fig. 18. Project E—Task E1, interior framed wind bent comparison

Project E Results

Task E1—Figure 18 shows comparative results for the east-west interior framed wind bent. Sizes were identical, except the LRFD story 10 column was one size smaller.

Task E2—Figure 19 shows AISC and LRFD sizes for the north-south trussed wind bent. Size variations were minor, except for the inner diagonals at story 2. The 40% larger LRFD size resulted from the more conservative LRFD provisions for slender compression members. Further review of this aspect of the LRFD Criteria is warranted.

Task E3—Figure 20 shows comparative sizes for the east-west exterior framed wind bent. Since the chosen wind drift criterion controlled all sizes, the results are identical.

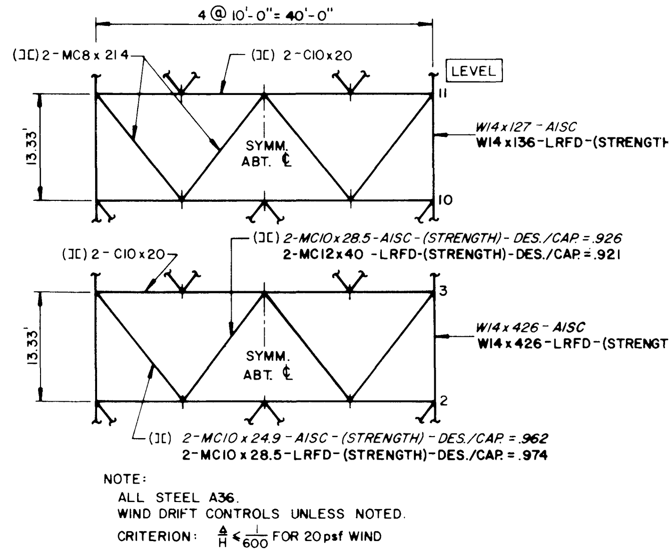
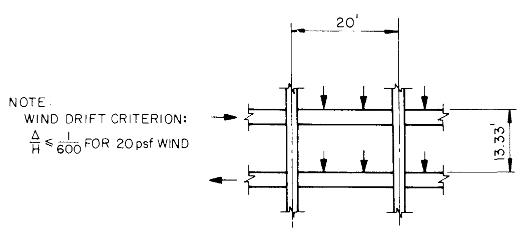


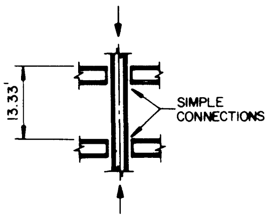
Fig. 19. Project E—Task E2, trussed wind bent comparison



STORY	10 - 11		2 - 3	
DESIGN METHOD	AISC	LRFD	AISC	LRFD
GIRDER SIZE*	*W24x61*	W24x61	*W30x99*	W30x99
CONTROLLED BY	*WIND DRIFT*	WIND DRIFT	*WIND DRIFT*	WIND DRIFT
DESIGN/CAPACITY RATIO	*0.494*	0.477	*0.426*	0.449
MIN. SIZE FOR STRENGTH	*W18x40*	W16x40	*W24x61*	W24x61
COLUMN SIZE*	*W14x74*	W14x74	*W14x158*	W14x158
CONTROLLED BY	*WIND DRIFT*	WIND DRIFT	*WIND DRIFT*	WIND DRIFT
DESIGN/CAPACITY RATIO	*0.590*	0.608	*0.663*	0.745

*ALL STEEL A36

Fig. 20. Project E—Task E3, exterior framed wind bent comparison



STORY	10-11		2-3	
DESIGN METHOD	*AISC*	LRFD	*AISC*	LRFD
COLUMN SIZE (ALL F_y = 50 ksi)	*W14x103*	W14x103	*W14x237*	W14x219
LL/DL RATIO	*0.379*	0.481	*0.308*	0.358
DESIGN/CAPACITY RATIO	*0.983*	0.930	*0.976*	0.966

Fig. 21. Project E—Task E3, interior column comparison (no wind)

The minimum girder size for strength alone is shown for information only. Figure 21 shows comparative sizes for the braced simple interior columns, which do not resist any wind in Task E3. LRFD produces either a smaller design/capacity ratio or a smaller column size.

DESIGNER'S EVALUATION OF LRFD

The four primary objectives of the Design Office Study were accomplished. For example, some revisions and clarifications were made in the LRFD Criteria. The acceptability of member sizes designed by LRFD was verified. Where differences between AISC and LRFD designs resulted, they could be logically explained. In some cases LRFD Criteria were modified, in others the AISC Specification provisions were in process of revision.

Using LRFD in the design office was not a difficult process, even without design aids, which are being developed. Applying LRFD does *not* require any complicated mathematics. After overcoming the initial hurdle of unfamiliarity, and with the help of design aids, designers will probably soon be using LRFD routinely in their offices.

Because of the calibration process used in developing LRFD, member sizes and complete building designs are not expected to be significantly different from those produced using the current AISC Specification. Where sizes differ, the LRFD size is likely to be smaller or lighter in many cases, especially when dead load is relatively large. Weight savings are likely to be no more than 5 or 10%, since LRFD was developed to provide a more uniform safety factor, rather than to produce large steel reductions at the expense of generally lowered safety.

Features of LRFD that were liked most by the writer are:

1. The 3-zone approach to computing buckling resistance of flexural members is good. Because there is a smooth transition zone between inelastic and elastic buckling, there are no discontinuities in bending capacity, as are now found in the AISC Specification.
2. Use of a lower load factor for dead load than for live load is more rational and produces results more in line with design codes for other structural materials, such as concrete.
3. There are improved future prospects of structural design using more unified design codes, based on maximum strength, for buildings of steel and other major structural materials. It would be desirable for the parts of these codes dealing with loads and load factors to be made identical.

The main advantage of the Load and Resistance Factor Design method, in this writer's opinion, is that an LRFD-designed building will have a more uniform safety margin, or structural reliability, throughout all its members.

REFERENCES

1. Proposed Criteria for Load and Resistance Factor Design of Steel Building Structures *Research Report 45, C.E. Dept., Washington University, St. Louis, Mo., May 1976.*
2. Specification for the Design, Fabrication and Erection of Structural Steel for Buildings *American Institute of Steel Construction, New York, N.Y., 1969, plus Supplements 1, 2 and 3.*